# CHAPTER 7.0
# SPREAD FOOTING DESIGN

The geotechnical design of a spread footing foundation is a two-part process. First the allowable soil bearing capacity must be established to insure stability of the footing and determine if the proposed structure loads can be supported on a reasonably sized footing. Second, the amount of settlement due to the actual structure loads must be predicted and time of occurrence estimated. Experience has shown that settlement is usually the controlling factor in the decision to use a spread footing foundation. This is not surprising as structural considerations usually limit tolerable settlements to values which can only be achieved on competent soils not prone to bearing capacity failure.

## 7.1 FOUNDATION DESIGN PROCEDURE

Foundation design is required for all structures to insure that the loads imposed on the underlying soil will not cause shear failures or damaging settlements. The duty of the foundation engineer is to establish the most economical design which safely conforms to prescribed structural criteria and properly accounts for the intended function of the structure. Essential to the foundation engineer's study is a rational method of design, whereby various foundation types are systematically considered and the optimum alternative selected. Indiscriminate selection of foundation type is verboten. Consideration of the following design approach will satisfactorily establish the proper type.

1. Determine the foundation loads to be supported and special constraints such as:

    a. Underclearance requirements which limit allowable total settlement.
    b. Structural design methodology which limits allowable differential settlement.
    c. Structural loads and tolerable deflections.
    d. Time constraints on construction.

    In general, a predesign discussion with the structural engineer will provide these answers and an indication of the degree of flexibility of the constraints.

2. Evaluate the subsurface data and laboratory testing with regard to reliability and completeness. The design method chosen should be commensurate with the quality and quantity of available geotechnical data, i.e., don't use state-of-the-art computerized analyses if you have not taken borings.

3. Consider alternate foundation types where applicable.

## 7.2 BEARING CAPACITY OF SPREAD FOOTINGS

Textbooks present varying theories and failure mechanisms for shallow footings. For the practicing engineer these theoretical discussions hold little interest. However, certain practical information can be drawn from the geometrics of the failure zone.

1. The bearing capacity of a footing is dependent on the strength of the soil within a depth below the footing of about 1 1/2 the footing width (unless much weaker soils exist just below this level).

Therefore, representative soil samples and frequent SPT values must be obtained in this zone. Continuous soil samples and SPT values should be routinely specified to a depth equal to twice the footing width. If the borings for a structure are done long before design, a good practice is to obtain continuous split spoon samples for the top 15 feet of each boring where footings may be placed on natural soil. The cost of this sampling is minimal but the knowledge gained is great including:

a. Thickness of existing topsoil.
b. Location of any thin zones of unsuitable material.
c. Accurate determination of depth of existing fill.
d. Improved ground water determination in the critical zone.
e. Representative samples in this critical zone to permit confident assessment of bearing capacity.

2. Often questions arise during excavation near existing footings as to the effect of soil removal on bearing capacity. In general, for weaker soils this zone extends outside the footing edge less than twice the footing width. Reductions in bearing capacity can be estimated by considering effects of removal within these zones. The lateral extent of this theoretical zone (Figure 7-1) is also useful in determining effects of ground irregularities on footing capacity or the effects of footing loads on adjacent facilities.

The general mechanism by which soils resist a footing load is similar to an embankment resisting shear failure. The load to cause failure must exceed the available soil strength on the failure plane and cause uplift of the weight of soil above the footing. When failure occurs the footing plunges into the ground and causes an uplift of soil adjacent to the sides of the footing.

The resistance to failure is based on the soil strength and amount of soil above the footing. The bearing capacity can be increased by:

1. Replacing or densifying the soil below the footing.

2. Increasing the embedment of the footing below ground.

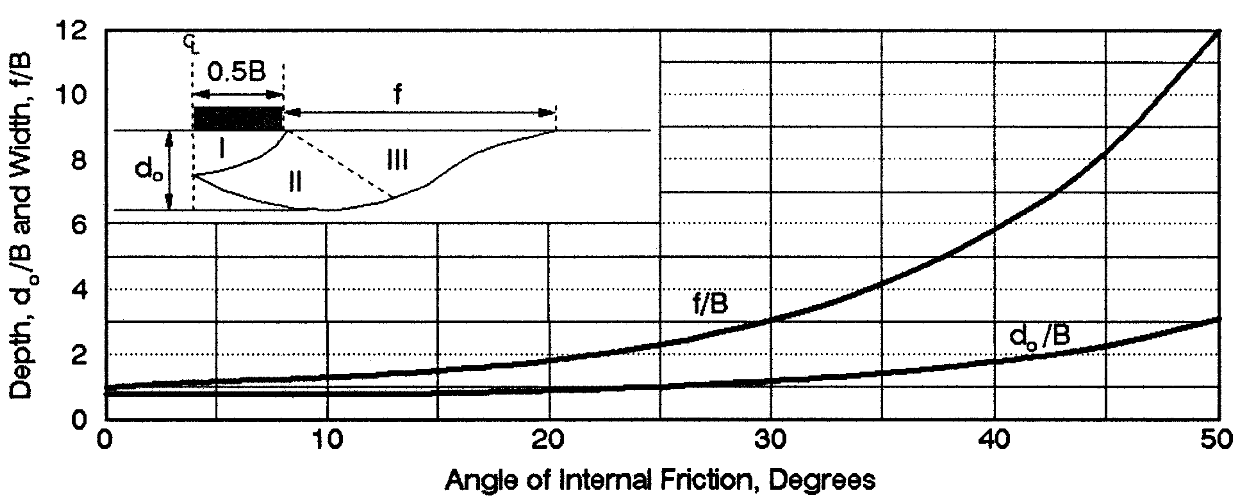



Figure 7-1: Variation of Depth ($d_0$) and Lateral Extent (f) of Influence of Footing with Angle of Friction

Common examples of improving bearing capacity are the support of temporary footings on pads of gravel or the embedment of mudsills a few feet below ground to support falsework. The design of these support systems is primarily done by bearing capacity analysis using the results of subsurface explorations and testing. Structural engineers who review falsework designs should carefully check the soil bearing capacity at foundation locations.

### 7.2.1 Bearing Capacity Computation

The procedure to be used to compute bearing capacity is as follows:

1. Review the structure plan to determine the proposed footing width. In the absence of data assume pier footing width equal to 1/3 the pier column height and abutment footing width equal to 1/2 the abutment height.

2. Review the soil profile to determine the position of the water table and the soil layer(s) which exist within the appropriate depth (1.5B) below the proposed footing level.

3. Review soil test data to determine the unit weight, friction angle and cohesion of the soils. In the absence of test data these values may be estimated for granular soils from standard penetration test data (Table 7-1) which has been corrected for overburden pressure. NOTE, the reliability of SPT values to determine shear strength of cohesive soils is poor. The SPT values in cohesive soils should not be used for determination of shear strengths for final design.

**TABLE 7-1**
**ESTIMATION OF SOIL PARAMETERS FROM STANDARD PENETRATION TESTS**

a. Granular Soil (Sand)

| **Description** | **Very Loose** | **Loose** | **Medium** | **Dense** | **Very Dense** |
|---|---|---|---|---|---|
| Standard penetration resistance corr'd, N'* | 0 | 4 | 10 | 30 | 50 |
| Approx. angle of internal friction, ($\phi$)degrees** | 25 – 30 | 27 – 32 | 30 – 35 | 35 – 40 | 38 – 43 |
| Approx. range of moist unit weight, ($\gamma$)pcf** | 70 – 100 | 90 – 115 | 110 – 130 | 120 – 140 | 130 – 150 |

* N' is SPT value corrected for overburden pressure.
** Use larger values for granular material with 5% or less fine sand and silt.

b. Cohesive soils (Clay) - (Rather unreliable, use only for preliminary estimate purposes).

| **Consistency** | **Very Soft** | **Soft** | **Medium** | **Stiff** | **Very Stiff** | **Hard** |
|---|---|---|---|---|---|---|
| $q_u$, ksf | 0 | 0.5 | 1.0 | 2.0 | 4.0 | 8.0 |
| Field standard penetration Resistance, N | 0 | 2 | 4 | 8 | 16 | 32 |
| $\gamma$(moist) pcf | 100 – 120 | 110 – 130 | | 120 – 140 | | |

4. Use the appropriate equation on Figures 7-2 through 7-5 to compute the ultimate bearing capacity. The continuous footing general case may be used when the footing length is 9 or more times the footing width. Also the bearing capacity factor $N_\gamma$ will usually be determined for a rough base condition since most footings are poured concrete. However the contact material smoothness must be considered for temporary footing such as wood grillages (rough), or steel supports (smooth) or plastic sheets (smooth). The safety factor for spread footing bearing capacity is selected both to limit the amount of soil strain and to account for variations in soil properties at footing locations.

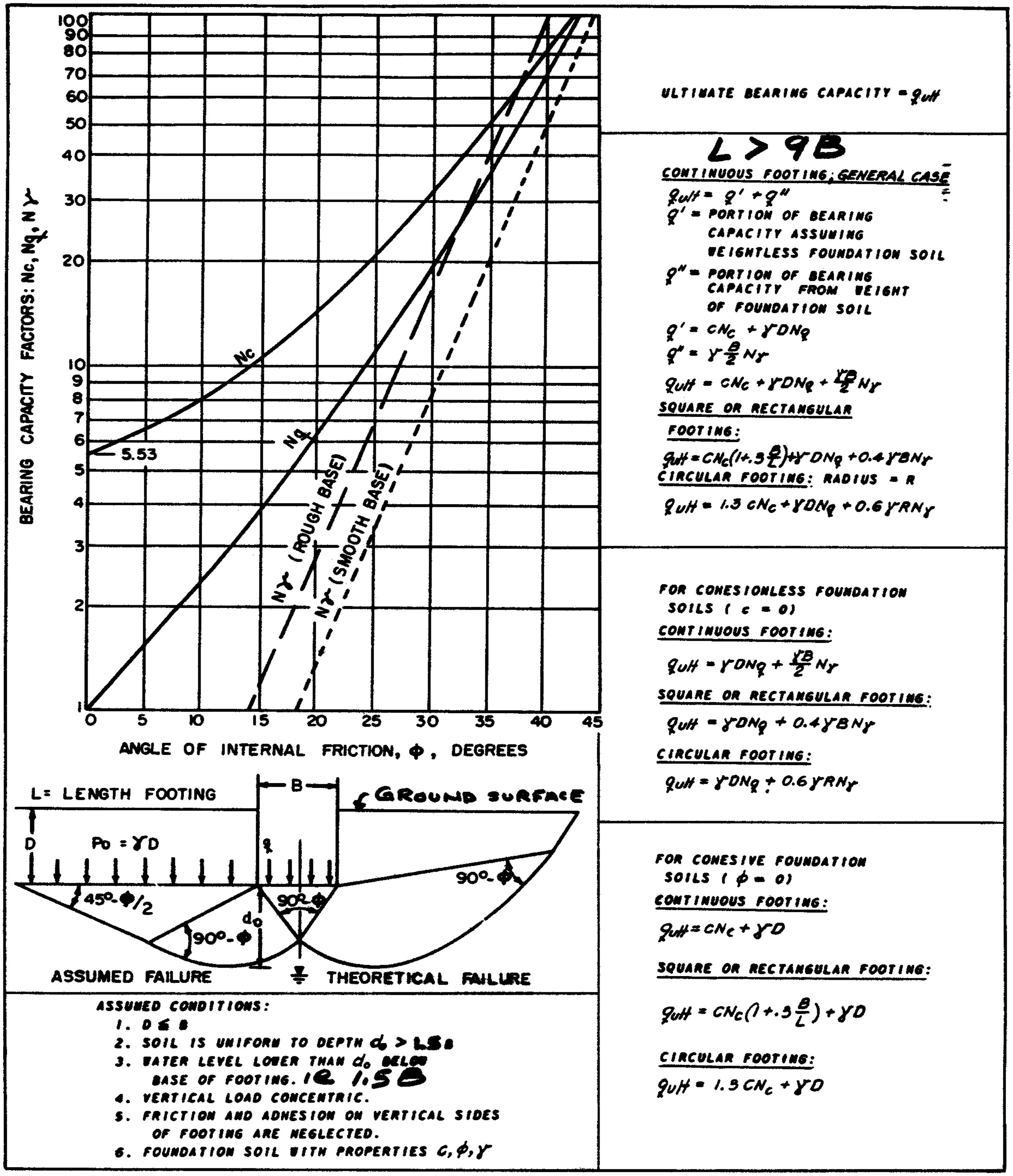
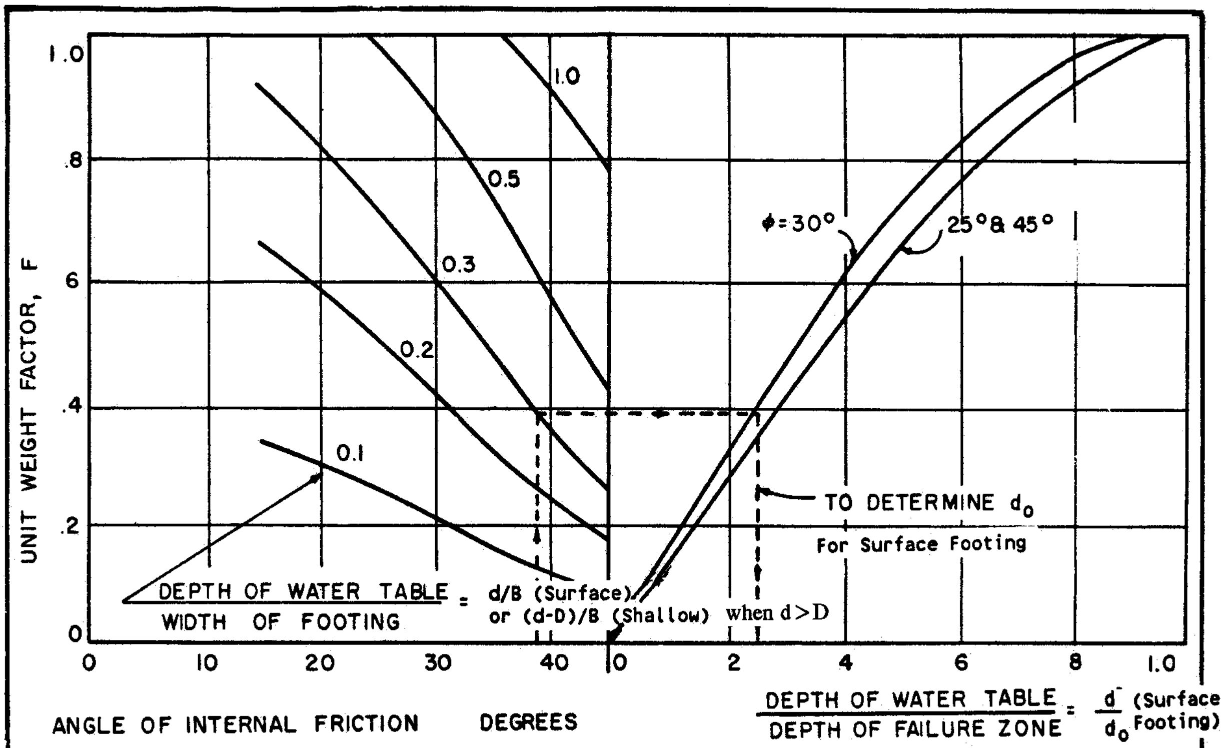
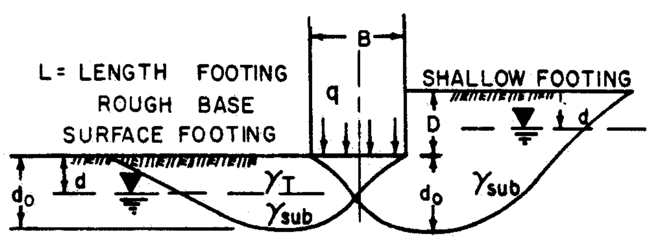



Figure 7-2: Ultimate Bearing Capacity of Shallow Footings with Concentric Loads




ASSUMED CONDITIONS:

1. GROUNDWATER LEVEL IS HORIZONTAL
2. PRESENCE OF GROUNDWATER HAS NO EFFECT ON COHESIVE SOIL WITH $\phi = 0$.

**CONTINUOUS FOOTING:**

**SURFACE FOOTING: D = 0**

$q_{ult} = cN_c + [\gamma_{sub} + F(\gamma_T - \gamma_{sub})]\frac{B}{2}N_\gamma$

**SHALLOW FOOTING: $D \leq B$**

IF $d \leq D$

$q_{ult} = cN_c + [\gamma_{sub}D + (\gamma_T - \gamma_{sub})d]N_q + 0.5\gamma_{sub}BN_\gamma$

IF $D < d \leq (D + d_0)$

$q_{ult} = cN_c + \gamma_T DN_q + [\gamma_{sub} + F(\gamma_T - \gamma_{sub})]\frac{B}{2}N_\gamma$

**VALUES OF BEARING CAPACITY FACTORS $N_c$ $N_q$ AND $N_\gamma$ ARE SHOWN IN FIGURE 14**

**RECTANGULAR FOOTING:**

**SURFACE FOOTING: D = 0**

$q_{ult} = cN_c(1 + .3\frac{B}{L}) + [\gamma_{sub} + F(\gamma - \gamma_{sub})]\,0.4BN_\gamma$

**SHALLOW FOOTING: $D \leq B$, IF $d \leq D$**

$q_{ult} = cN_c(1 + .3\frac{B}{L}) + [\gamma_{sub}D + (\gamma_T - \gamma_{sub})d]N_q + 0.4\gamma_{sub}BN_\gamma$

IF $D < d \leq (D + d_0)$

$q_{ult} = cN_c(1 + .3\frac{B}{L}) + \gamma_T DN_q + [\gamma_{sub} + F(\gamma_T - \gamma_{sub})]\,0.4BN_\gamma$

**CIRCULAR FOOTING: RADIUS = R = B/2**

**SURFACE FOOTING: D = 0**

$q_{ult} = 1.3cN_c + [\gamma_{sub} + F(\gamma_T - \gamma_{sub})]\,0.6RN_\gamma$

**SHALLOW FOOTING: $D \leq 2R$, IF $d \leq D$**

$q_{ult} = 1.3cN_c + [\gamma_{sub}D + (\gamma_T - \gamma_{sub})d]N_q + [0.6\gamma_{sub} \cdot RN_\gamma]$

IF $D < d \leq (D + d_0)$

$q_{ult} = 1.3cN_c + \gamma_T DN_q + [\gamma_{sub} + F(\gamma_T - \gamma_{sub})]\,0.6RN_\gamma$

Figure 7-3: Ultimate Bearing Capacity with Ground Water Effect

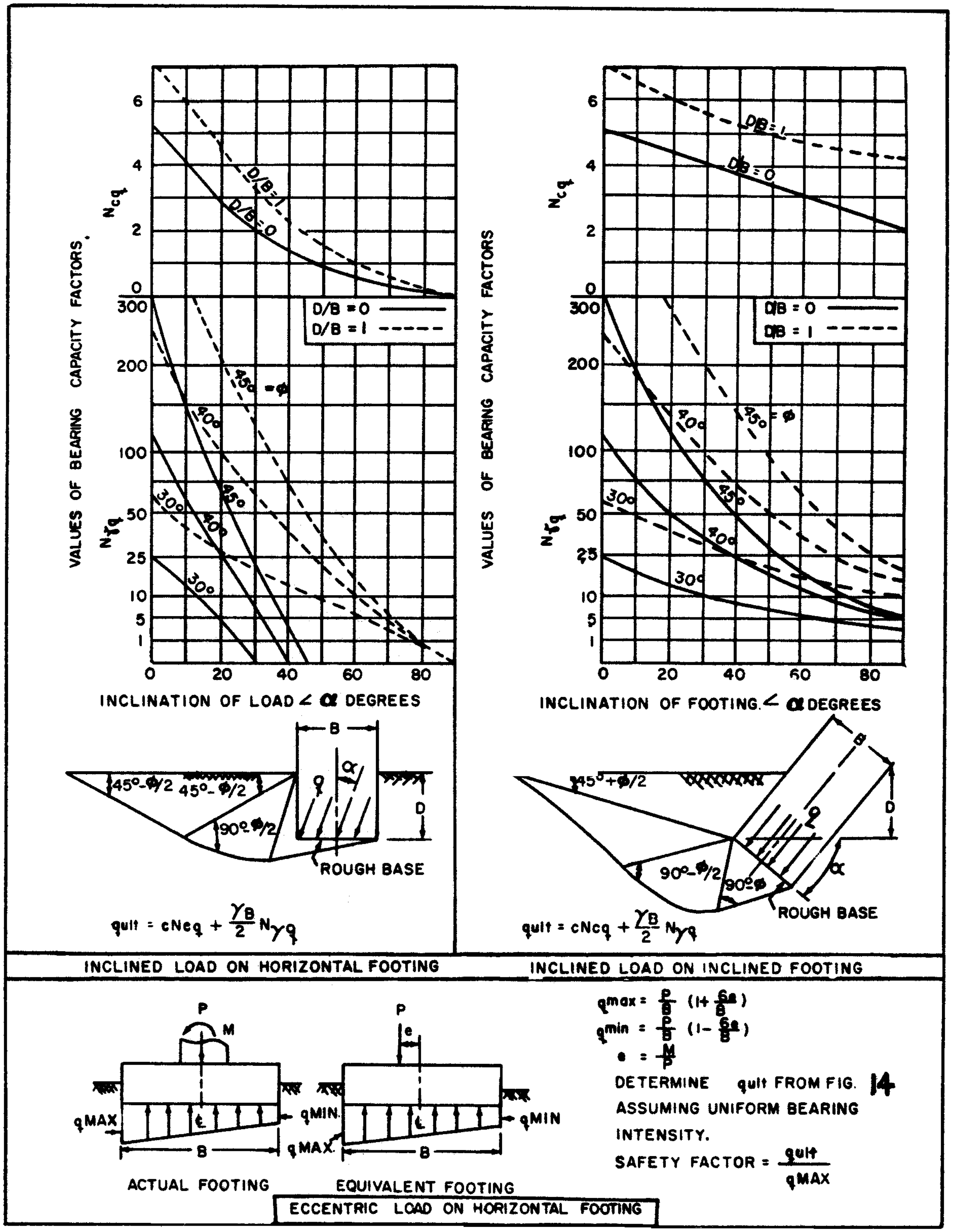
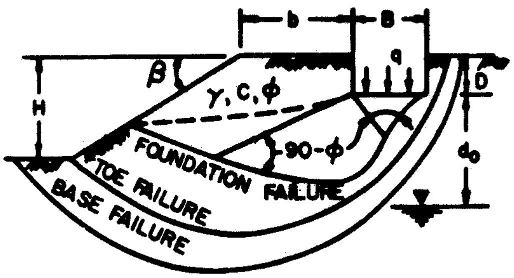



Figure 7-4: Ultimate Bearing Capacity Continuous Footing with Eccentric or Inclined Loads

## CASE I: CONTINUOUS FOOTING AT TOP OF SLOPE




Water at $d_o \geq B$

$$q_{ult} = cN_{cq} + \gamma_T \frac{B}{2} N_{\gamma q} \quad (1)$$

Water at Ground Surface

$$q_{ult} = cN_{cq} + \gamma_{sub} \frac{B}{2} N_{\gamma q} \quad (2)$$

If $B \leq H$:

Obtain $N_{cq}$ from Figure 17B for Case I with $N_o = 0$.
Interpolate for values of $0 < D/B < 1$

Interpolate $q_{ult}$ between EQ (1) and (2) for water at intermediate level between ground surface and $d_o = B$.

If $B > H$:

Obtain $N_{cq}$ from Figure 17B for Case I with stability number

$$N_o = \frac{\gamma H}{C}.$$

Interpolate for values $0 < D/B < 1$ for $0 < N_o < 1$. If $N_o \geq 1$, stability of slope controls ultimate bearing pressure.

Interpolate $q_{ult}$ between EQ (1) and (2) for water at intermediate level between ground surface and $d_o = B$. For water at ground surface and sudden drawdown: substitute $\phi'$ for $\phi$ in EQ (2)

$$\phi' = \tan^{-1}\left(\frac{\gamma_{sub}}{\gamma_T} \tan \phi\right)$$

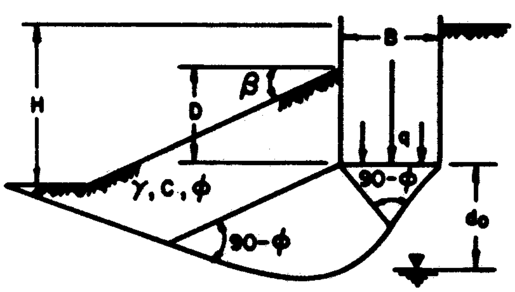
Cohesive soil ($\phi = 0$)

Substitute in EQ (1) and (2) D for $B/2$ and $N_{\gamma q} = 1$.

Rectangular, square or circular footing:

$$q_{ult} = \left[q_{ult} \text{ for continuous footing as given above}\right] \times \left[\frac{q_{ult} \text{ for finite footing}}{q_{ult} \text{ for continuous footing}}\right] \text{ from Fig. 14}$$

## CASE II: CONTINUOUS FOOTINGS ON SLOPE




Same criteria as for Case I except that $N_{cq}$ and $N_{\gamma q}$ are obtained from diagrams for Case II

Figure 7-5A: Ultimate Bearing Capacity for Shallow Footing Placed on or Near a Slope

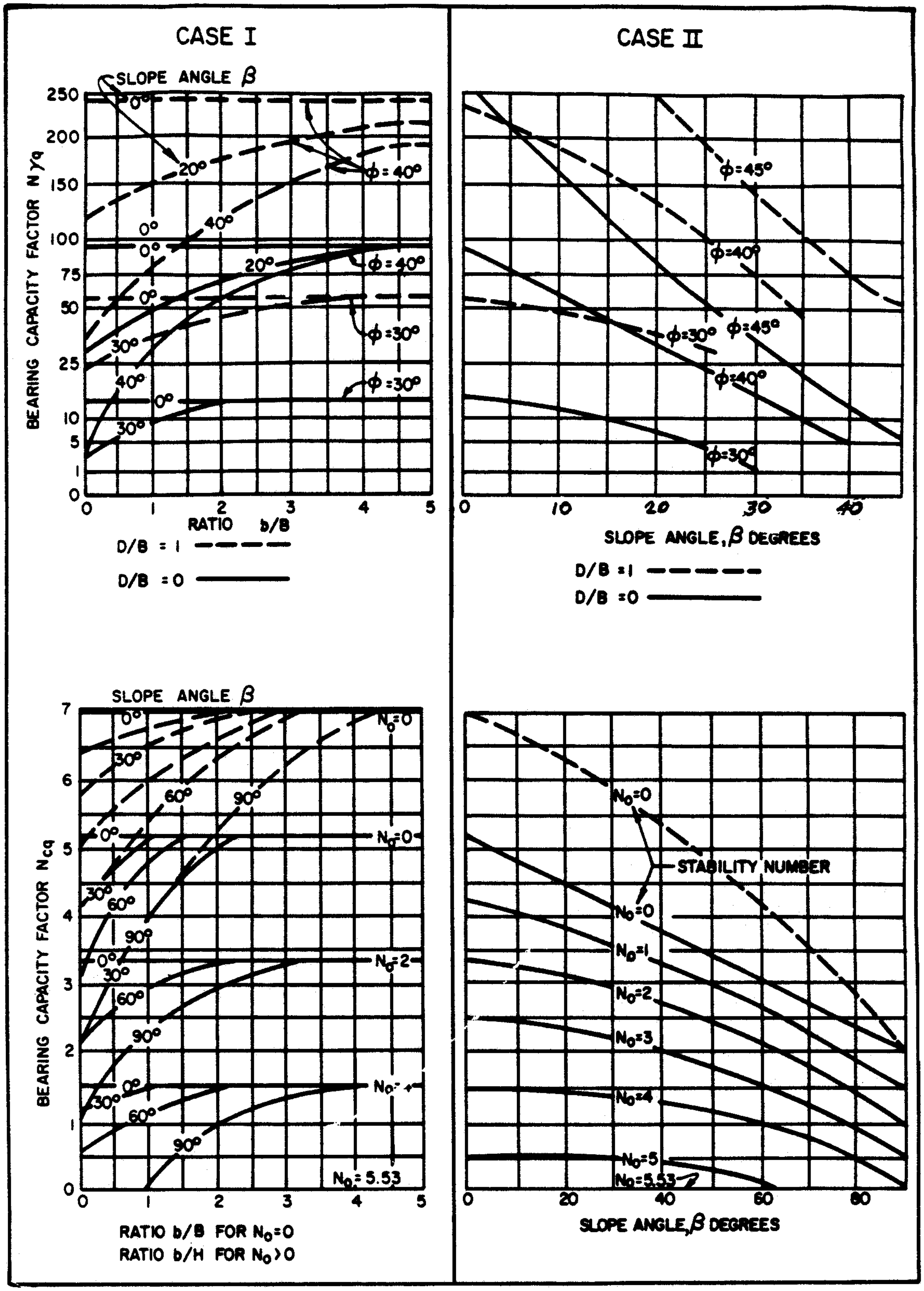



Figure 7-5B: Bearing Capacity Factors for Shallow Footing Placed on or Near A Slope

### 7.2.2 Practical Aspects of Bearing Capacity Computations

Many footings are designed structurally to resist a combination of vertical and horizontal loads which produce a trapezoidal load distribution under the footing. In sizing a footing to accommodate a recommended maximum allowable bearing capacity, the question arises on whether the recommended value refers to the average value across the footing or the maximum value under the footing edge. Satisfactory results may be obtained by sizing the footing so that the average pressure does not exceed the recommended value and the maximum edge pressure does not exceed 1.3 times the average value.

The effect of a high ground water table on the bearing capacity of a footing is frequently over-estimated. Some textbooks and public codes mandate reduction of the allowable bearing capacity by one half if the ground water is within a depth of one footing width below the footing. Such a large reduction only applies if the design ground water level is at or above ground surface in granular soils. The only soil element affected by ground water is the unit weight ($\gamma$). An examination of the general bearing capacity equation indicates two of the three terms include soil unit weight. One term refers to the amount of soil above the footing; the other to soil below the footing. As water rises up toward the footing level, only one factor is reduced. For complete reduction by half, the water must rise above the ground. The effect of high ground water on bearing capacity is accurately taken into account in Figure 7-3.

The general effects of changes in either soil properties or footing dimensions on bearing capacity need to be understood. The general equation for bearing capacity is:

$$q_{ult} = cN_c + \gamma DN_q + 1/2\gamma BN_\gamma \tag{7-1}$$

Note first that bearing capacity is composed of separate contributions from the soil's cohesive strength, the embedment depth of the footing, and the soil's frictional strength. Table 7-2 shows how bearing capacity can vary with changes in physical properties or dimensions.

**TABLE 7-2**
**VARIATION IN BEARING CAPACITY WITH CHANGES IN PHYSICAL PROPERTIES OR DIMENSIONS**

| Properties and Dimensions | Cohesive Soil | Cohesionless Soil |
|---|---|---|
| $\gamma$ = Unit Weight<br>D = Footing Embedment<br>B = Footing Width | $\phi = 0$<br>c = 1000 psf<br>$q_{ult}$ (psf) | $\phi = 30^o$<br>c = 0<br>$q_{ult}$ (psf) |
| A. Initial situation $\gamma$ = 120 pcf, D = 0', B = 5' Deep water table | 5530 | 5400 |
| B. Effect of embedment D = 5', $\gamma$ = 120 pcf, B = 5', deep water table | 6130 | 17400 |
| C. Effect of width, B = 10' $\gamma$ = 120 pcf, D = 0', deep water table | 5530 | 10800 |
| D. Effect of water table at surface $\gamma$ = 57.6 pcf, D = 0', B = 5' | 5530 | 2592 |

Notice that the effect of the variables on the bearing capacity in cohesive soils is minimal. Only the embedment has an effect on bearing capacity. Also note that the water table rise does not influence cohesion. Interparticle bonding will remain unchanged unless the clay contains minerals which react to water immersion, i.e., expansive minerals, or the clay is reworked.

Notice the effect on cohesionless soils is great when properties and dimensions are changed. The embedment effect is particularly important. Removal of soil from over an embedded footing, either by excavation or scour, can substantially reduce bearing capacity and cause footing subsidence. Rehabilitation or repair of existing spread footing often requires excavation of the soil above the footing. If the effect of this removal on bearing capacity is not considered, the footing may move downward; resulting in structural distress.

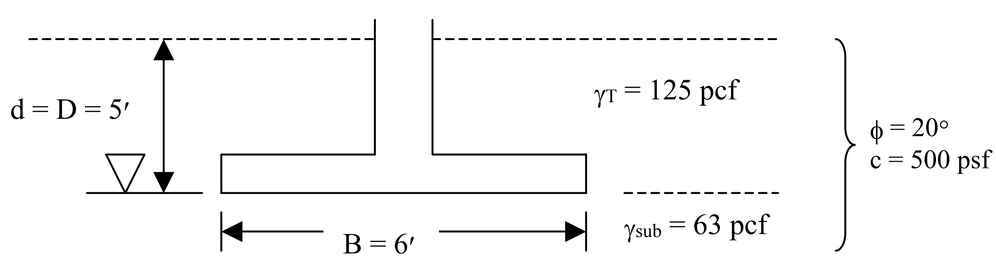
Two modes of bearing capacity failure exist: general shear failure and local shear failure. Local shear failure is characterized by a "punching" of the footing into the ground when weak soils exist below footing level or when very narrow footings are used. This local condition seldom applies to bridge structures because spread footing are not used on obviously weak soils and relatively large footing sizes are needed for structural stability.

The mechanism of general bearing capacity failure is similar to the embankment failure mechanism. However, the footing analysis is a 3-dimensional analysis as opposed to the 2-dimensional slope stability analysis. The bearing capacity factors $N_c$, $N_q$ and $N_\gamma$ relate to the actual volume of soil involved in the failure. A cursory study of the footing failure cross section in Figure 7-2, discloses that the depth and lateral extent of the failure (and therefore the value of $N_c$, $N_q$ and $N_\gamma$) is determined by the dimensions of the wedge-shaped zone directly below the footing. As the friction angle increases, the depth and width of the failure zone increase; thus mobilizing more soil shear strength and increasing the bearing capacity.

Substantial downward movement of the footing is required to completely mobilize the shearing resistance along the entire failure surface. For this reason the safety factor which is used to find allowable bearing capacity is composed of two partial safety factors; a factor of 2.0 to limit strain and a factor of 1.25 - 1.50 for uncertainties in soil information. The total safety factor is usually 3 if standard penetration values were used to determine strength properties of the soil. This large safety factor insures that only minimal movement (strain) is necessary to fully mobilize the allowable bearing capacity.

Lastly, in reporting the results of bearing capacity analyses, always include the footing width that was used to compute the bearing capacity. Most often the geotechnical engineer must assume a footing width as bearing capacity analyses are completed before structural design begins. It is recommended that bearing capacity be computed for a range of possible footing widths and those values be included in the foundation report with a note stating that if other footing widths are used, the geotechnical engineer should be contacted. Remember that changes in footing width cause large changes in unit bearing capacity in granular soils.

**Example 7-1**: Determine the Allowable Bearing Capacity for a Rough Base Square Footing Using a Safety Factor of 3.



**Solution:**

Assuming a general shear condition, enter the bearing capacity chart for $\phi = 20°$ and read $N_c = 14$, $N_q = 6$, $N\gamma = 3$. Also note that formula for bearing capacity must account for the square footing and the water table within the failure zone.

$$q_{ult} = (1 + 0.3\frac{B}{L})cN_c + [\gamma_{sub} D + (\gamma_T - \gamma_{sub})d]N_q + 0.4\gamma_{sub} BN_\gamma \qquad (7\text{-}2)$$

$$= (1.3)(500)14 + [63(5) + (125 - 63)5]6 + 0.4(63)(6)(3)$$

$$= 9100 + 3750 + 450$$

$$q_{ult} = 13{,}300 \text{ psf}$$

$$q_{all} = \frac{q_{ult}}{3} = \frac{13{,}300}{3} \cong 4{,}430 \text{ psf}$$

### 7.2.3 Spread Footing Load Tests

Spread footing load tests can be used to verify both bearing capacity and settlement predictions. Full scale tests have been done on predominantly granular soils. An example is the I-359 project in Tuscaloosa, Alabama where dead load was placed on 12' x 12' footings to create a foundation contact pressure of over 4 tsf. The greatest settlement recorded was about 0.1 inches. (An additional 0.1 inch was recorded when the footing concrete was placed).

A new dynamic procedure called the WAK test, is also available to assess the stiffness of soils below footings (ASCE Journal of Geotechnical Engineering, Vol. 116, No. 3, March 1990).

### 7.2.4 Computer Program

FHWA has funded the development of a user friendly computer program, CBEAR. The users manual is FHWA-TA-91-047, "CBEAR - Bearing Capacity Analysis of Shallow Foundations." The major use of this program is to compute bearing capacity of footings with complex loading conditions.

## 7.3 SETTLEMENT OF SPREAD FOOTINGS

The controlling factor in the design of a spread footing foundation is usually tolerable settlement. Prediction of settlement may be routinely accomplished with adequate geotechnical data and a knowledge of the proposed structure loads. The accuracy of the prediction is only as good as the quality of the geotechnical data and the estimation of the actual loads placed on the footing. Settlements of spread footings are frequently overestimated by engineers for the following reasons:

1. The structural load (P) causing the settlement is overestimated. In the absence of actual structural loads, geotechnical engineers conservatively assume that the footing pressure equals the maximum allowable soil bearing pressure.

2. Settlement occurring during construction is not subtracted from total predicted amounts.

3. Preconsolidation of the subsoil is not accounted for in the analysis. This preconsolidation may be due to a geologic load applied in past time or to removal of significant amounts of soil in construction previous to placing the foundation. This error can cause a grossly overestimated settlement.

To rationally predict settlement of spread footings, the following procedure should be used:

### 7.3.1 General Procedures for Both Cohesionless and Cohesive Soils

1. Plot soil profile including soil unit weights, consolidation test values for design and SPT results (N).

2. Draw existing effective overburden pressure diagram ($P_o$) with depth.

3. Plot design bearing pressure on $P_o$ diagram at proper footing level.

4. Distribute design bearing pressure with depth by 2 on 1 method or other appropriate distribution method.

   2:1 Pressure Distribution Method:

   If footing is continuous, $L \geq 9W$; $\Delta P = \dfrac{W}{W+X}(P)$ (7-3a)

   If footing is rectangular, $L < 9W$; $\Delta P = \dfrac{WL}{(W+X)(L+X)}(P)$ (7-3b)

   Where: X = depth below footing
   W = footing width
   L = footing length
   P = applied footing pressure

   a. Project 2 vertical on 1 horizontal lines down from footing corners. Compare original footing area to area generated on a plane at various depths below the footing, i.e., if a 10′ by 40′ footing is loaded to 2000 psf, the pressure in the ground at 20′ below footing level is:

$$\Delta P = \frac{10 \times 40\ (2000)}{(10+20)\ (40+20)} = 444 \text{ psf}$$

5. Extend footing pressure distribution at least to a level where the distributed footing pressure (ΔP) is 1/10 of the overburden pressure at that depth. This depth is commonly referred to as the critical depth.

### 7.3.2 Settlement Computation for Cohesionless Soils

Settlement of granular soils is usually elastic and consolidation occurs immediately on application of load.

1. Determine corrected SPT value (N′) from Figure 6-5.

2. Determine bearing capacity index (C′) by entering Figure 6-6 with N′ value from (1).

3. Compute settlement in 10′ ± increments of depth from

$$\Delta H = H \frac{(1)}{C}, Log \frac{P_0 + \Delta P}{P_0} \tag{6-1}$$

Where:

| | | | |
|---|---|---|---|
| | $\Delta H$ | = | Settlement |
| | H | = | Thickness of soil layer considered |
| | C′ | = | Bearing capacity index |
| | $P_o$ | = | Existing effective overburden pressure at center of considered layer (psf) |
| | $\Delta P$ | = | Distributed footing pressure at center of considered layer (psf) |
| | $P_F$ | = | $P_o + \Delta P$ |

4. Studies conducted by FHWA indicated that this procedure is conservative and will over-predict the settlement by a factor of about 2.

### 7.3.3 Engineering Practice - Settlement and Differential Settlement (see publication FHWA-RD-86-185 for details)

A common practice for predicting settlement of footings on sand is to use one or more of the available calculation methods i.e., Hough, Peck-Bazaraa, D'Appolonia, Schmertmann. Engineering judgment is then used to select one of the results, or average the results, based on the appropriate approach. Experience has shown that structure foundations consisting of footings designed in this manner have a very high probability of acceptable performance.

A practical method for calculating differential settlement between adjacent footings on sand involves one or more of the following concepts:

1. If borings are performed at each footing location, calculate the differential settlement as the difference in the estimated total settlement of each footing, calculated based on the individual borings.

2. Lesser amounts of boring data only permit empirical estimates such as suggested by Terzaghi and Peck (1967), if footings are about the same length and width, calculate maximum differential settlement as 50 percent of the maximum total settlement. If footings are of different sizes, calculate differential as 75 percent of the maximum total value.

3. If the penetration resistance of the soil is highly variable from boring to boring, calculate maximum differential settlement as 100 percent of the maximum total settlement.

### 7.3.4 Settlement Computation for Cohesive Soils

Settlement of spread footings on cohesive soils is usually due to primary compression as spread footings are usually not placed on soils with significant secondary compression characteristics.

1. Analyze consolidation test data to determine:

   a. Preconsolidation pressure ($P_c$)

   b. Initial void ratio ($e_o$) at $P_o$

   c. Compression and recompression indices ($C_c$ & $C_r$)

1. (ALT) In the absence of consolidation test data, settlement may be approximated from Atterberg limit and moisture content data as follows in (a) through (c). This method is only recommended for use in design for soils not conducive to consolidation testing.

   a. Soil may be assumed to be preconsolidated to pressures above typical loads if the liquidity index ([moisture content minus plastic limit] divided by plastic index) is less than 0.7.

   b. Initial void ratio is determined for saturated soils by multiplying the specific gravity times the moisture content divided by 100.

   c. $C_c$ and $C_r$ are determined by dividing the moisture content by 100 and 1000 respectively.

2. Compute settlement in 10′ ± increments of depth or at soil layer boundaries from

$$\Delta H = H \frac{C_c}{1+e_0} \text{Log} \frac{P_F}{P_0} \qquad \text{(For normally consolidated soils only)} \qquad (6\text{-}2a)$$

3. Compute time for settlement from $t = (TH_v^2) / C_v$, following the procedure shown for embankment settlement in the previous chapter.

These settlement analyses may be varied by the foundation engineer to determine:

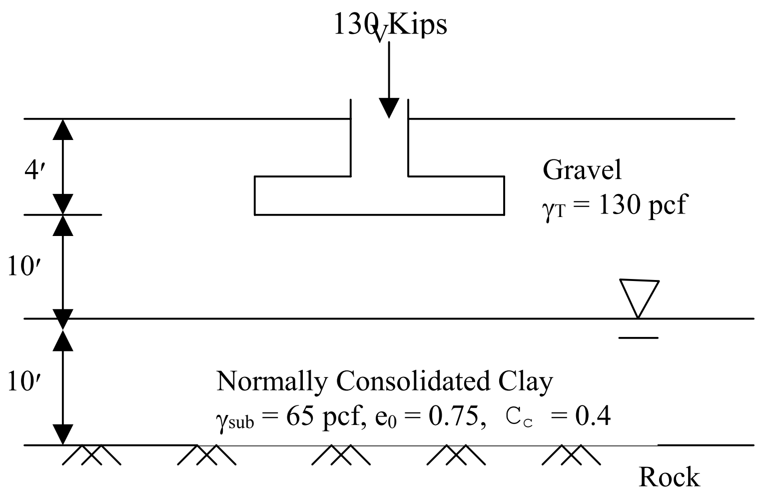
1. Percentage of settlement due to dead and live loads.

2. Effects of adjacent fill placement.

3. Footing width required to limit settlement to a tolerable value.

4. Amount of preload needed to reduce subsequent structure settlement.

**Example 7-2:** Determine the Settlement Of the 10′ × 10′ Square Footing Due To A 130 Kip Axial Load. Assume The Gravel Layer Is Incompressible.




**Solution:**

Find Overburden Pressure, $P_0$, at center of Clay Layer
$P_0 = (14' \times 130 \text{ pcf}) + (5' \times 65 \text{ pcf}) = 2{,}145 \text{ psf}$

Find Change in Pressure ()P) at Center of Clay Layer Due to Applied Load.

$$\Delta P = \frac{130\,\text{Kips}}{(10+15)^2} = \frac{130\text{Kips}}{625} = 208\,\text{psf}$$

Find Settlement

$$\Delta H = H\frac{C_c}{1+e_0}\text{Log}\frac{P_0+\Delta P}{P_0} \tag{6-2}$$

$$= 10\left(\frac{0.4}{1+0.75}\right)\text{Log}\frac{2145\,\text{psf}+208\,\text{psf}}{2145\,\text{psf}}$$

$$\Delta H = 0.09' = 1.1''$$

## 7.4 SPREAD FOOTINGS ON EMBANKMENTS

One of the most basic conclusions established by foundation engineers was the desirability of placing footings on controlled fills. In general, the fill weight is many times the imposed footing load. If adequate time is allowed for the foundation soil to consolidate under the fill load, subsequent application of the smaller structure load will result in negligible structure settlement. In bridge construction, common practice is to build the approach embankment excluding the area to be occupied by the abutment and allow settlement to occur prior to abutment construction.

Field evaluation of spread footings placed in compacted embankments, constructed of select granular material, have shown that spread footings will provide satisfactory performance. A 1978 performance evaluation (FHWA RD-81/184) was conducted through a joint study between FHWA and the Washington State Highway Department. A visual inspection was made of the structural condition of 148 highway bridges supported by spread footings on compacted fill throughout the State of Washington. The approach pavements and other bridge appurtenances were also inspected for damage or distress that could be attributed to the use of spread footings on compacted fill. This review, in conjunction with detailed survey investigations of the foundation movement of 28 selected bridges, was used to evaluate the performance of spread footings on compacted fills. The study concluded that spread footings can provide a satisfactory alternative to piles especially when high embankments of good quality borrow materials are constructed over satisfactory foundation soils. None of the bridges investigated displayed any safety problems or serious functional distress. All bridges were in good condition. In addition to the performance evaluation, cost analyses and tolerable movement correlation studies were made to further substantiate the feasibility of using spread footings in lieu of expensive deep foundation systems. Cost analyses showed spread footings were 50 to 65 percent cheaper than the alternate choice of pile foundations. Foundation movement studies showed that these bridges have easily tolerated differential settlements of 1 to 3 inches without serious distress. A second nationwide study of 314 bridges (FHWA RD-85/107) arrived at similar conclusions. Unfortunately many agencies continue to disregard spread footing foundation alternates for highway structures. In NCHRP Synthesis 107 "Shallow Foundations For Highway Structures" the author concisely summarizes the chapter on performance criteria as follows:

"It is very clear that the tolerable settlement criteria currently used by most transportation agencies are extremely conservative and are needlessly restricting the use of spread footings for bridge foundations on many soils. Angular distortions of 1/250 of the span length and differential vertical movements of 2 to 4 inches, depending on span length, appear to be acceptable, assuming that approach slabs or other provisions are made to minimize the effects of any differential movements between abutments and approach embankments. Finally, horizontal movements in excess of 2 inches appear likely to cause structural distress. The potential for horizontal movements of abutments and piers should be considered more carefully than is done in current practice."

## 7.5 APPLE FREEWAY DESIGN EXAMPLE – SPREAD FOOTING DESIGN

In this chapter the Apple Freeway is used to illustrate the design process for spread footings for the pier and abutment. The computation process for evaluation of bearing capacity and settlement analysis are presented.

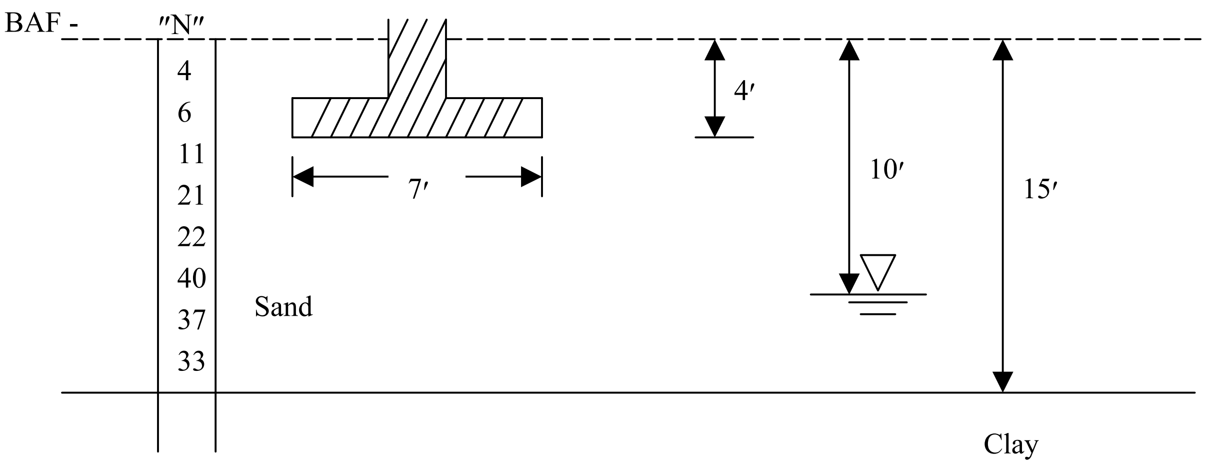
| | | |
|---|---|---|
| Site Exploration | Terrain Reconnaissance<br>Site Inspection<br>Subsurface Borings | |
| Basic Soil Properties | Visual Description<br>Classification Tests<br>Soil Profile | |
| Laboratory Testing | Po Diagram<br>Test Request<br>Consolidation Results<br>Strength Results | |
| Slope Stability | Design Soil Profile<br>Circular Arc<br>Analysis Sliding Block<br>Analysis Lateral Squeeze | |
| Embankment Settlement | Design Soil Profile<br>Settlement<br>Time – Rate<br>Surcharge<br>Vertical Drains | |
| → | **Spread Footing Design** | Design Soil Profile<br>Pier Bearing Capacity<br>Pier Settlement<br>Abutment Settlement<br>Vertical Drains<br>Surcharge |
| Pile Design | Design Soil Profile<br>Static Analysis – Pier<br>Pipe Pile<br>H – Pile<br>Static Analysis – abutment<br>Pipe Pile<br>H – Pile<br>Driving Resistance<br>Abutment Lateral Movement | |
| Construction Monitoring | Wave Equation<br>Hammer Approval<br>Embankment Instrumentation | |

Apple Freeway Design Example – Spread Footing Design
Exhibit A

**Pier Footing**

**Given:** The footing geometry and subsurface condition shown below.

**Required:** Compute the allowable bearing capacity, anticipated settlement and settlement rates for the pier footing.




Assumptions:

- Footing embeded 4′ below ground
- Footing width = 1/3 pier height = 7′
- Footing length = 100′
  L/W = 100/7 > 9 ∴ Continuous
- Water level 6′ below Footing (< 1.5B)
- Use SPT values to find $\phi$

## Compute Allowable Bearing Capacity

**Step 1: Find N′ below footing (use Figure 6-5 to obtain N′/N).**

| Depth | $P_0$ (psf) | N (bpf) | N′/N | N′ |
|---|---|---|---|---|
| 5 | 550 | 11 | 1.9 | 21 |
| 7 | 770 | 21 | 1.55 | 33 |
| 8 | 880 | 22 | 1.45 | 32 |
| 10 | 1100 | 40 | 1.27 | 51 |
| 12 | 1195 | 37 | 1.19 | 44 |
| 14 | 1290 | 33 | 1.12 | 37 |
| | | | Avg. N′ = 36 ∴ $\phi \approx 36°$ | |

**Step 2: Determine ultimate capacity ($Q_{ult}$).**

$$Q_{ult} = \cancelto{0}{cN_c} + \gamma_T DN_q + [\gamma_{sub} + F(\gamma_T - \gamma_{Sub})]\ \frac{B}{2} N\gamma$$

$$= (110)(4)(40) + [47.6 + (0.9)(62.4)]\left(\frac{B}{2}\right) 50 \qquad (N_q \ \&\ N_\gamma \text{ from Figure 7-2})$$

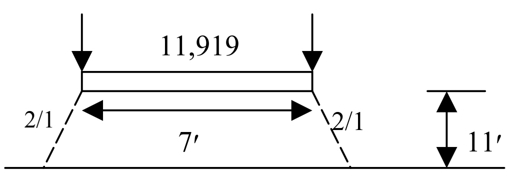
$$F@\frac{d - D}{B} \text{ of } 0.86 = 0.9 \ \ \text{(Figure 7-3)}$$

$Q_{ult}$ = 17,600 + 18,158 = 35,758 psf

**Step 3: Determine allowable bearing capacity (use F.S. = 3)**

$$Q_{all} = \frac{35{,}758}{3} = 11{,}919 \text{ psf or} \sim 6 \text{ tsf}$$

**CHECK PRESSURE TRANSMITTED TO CLAY LAYER VERSUS ALLOWABLE CLAY BEARING CAPACITY**




Clay Layer

**Step 1: Determine pressure on clay layer**

$$\text{Pressure @ clay surface} = \left(\frac{7}{7+11}\right)(11{,}919\ psf)$$

$P_{clay}$ = 4,635 psf

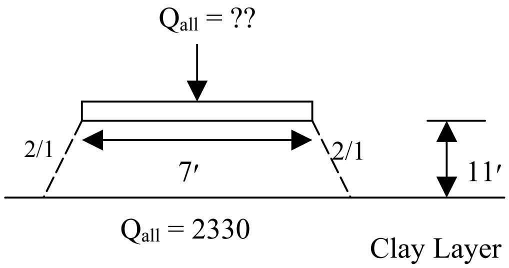
**Step 2: Check pier bearing capacity**

Pier Bearing Capacity at Top of Clay Layer

$$Q_{ult.clay} = cN_c + P_0N_q$$

$$= (1100)(5.14) + (1338)(1) = 6992 \text{ psf}$$

$$Q_{all\ clay} = \frac{6992}{3} = 2330 \text{ psf}$$




Need to reduce footing pressure as $Q_{all\ clay} < P_{clay}$

$$Q_{all\ max} = Q_{all\ clay}\ \left(\frac{7+11}{7}\right)$$

$$Q_{all\ max} = 2330\ \left(\frac{18}{7}\right) = 5990\ \text{psf}$$

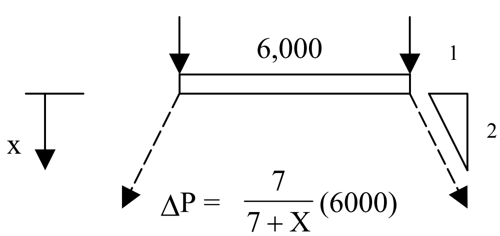
$Q_{all\ max}$ = 5,990 psf $\cong$ 3 tsf

Check w/ bridge designer to see if 3 tsf is a realistic pressure. Designer estimates a max. structure load of 2200 tons, and a minimum footing width of 7′ $\therefore$ Est. footing pressure (max) = $\frac{2200}{7 \times 100}$

$Q_{FTS} \cong 3.1$ tsf
Use Q = 3 tsf for Settlement Analysis

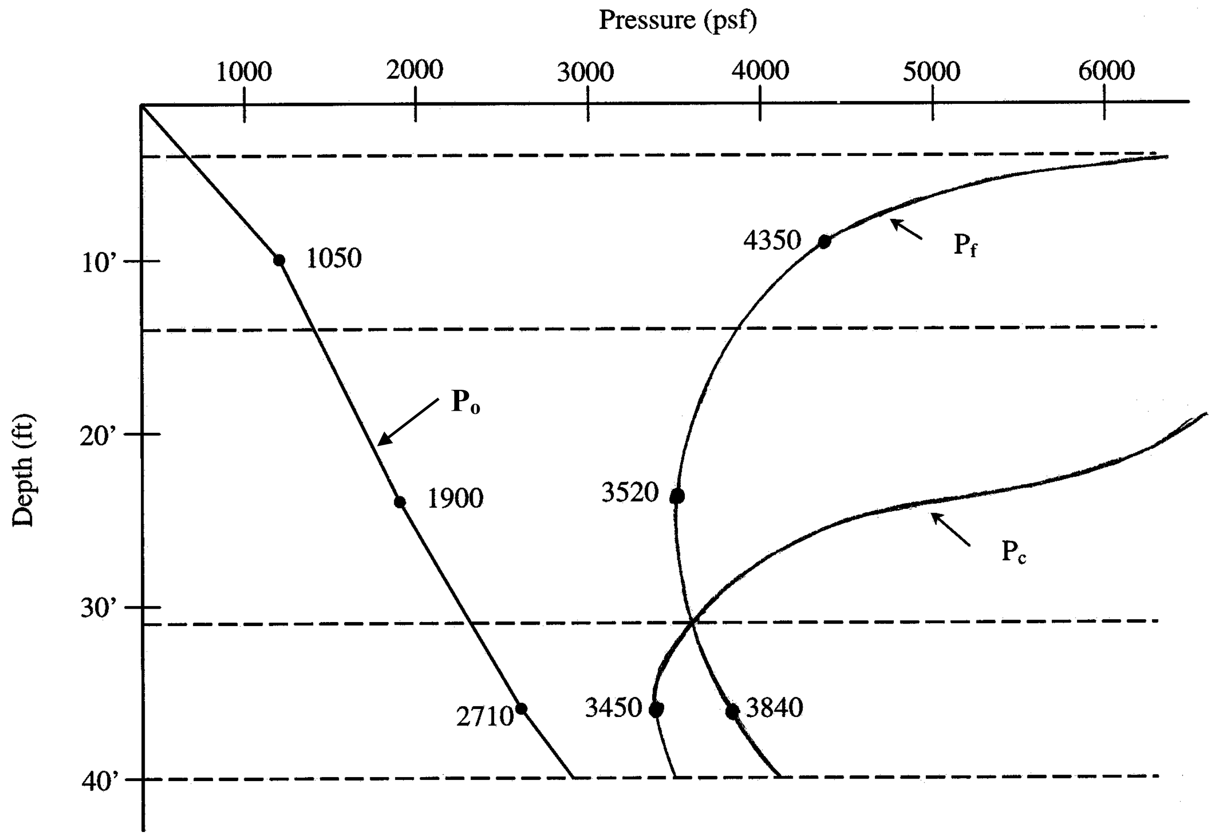
**Check Settlement**

**Step 1: Find pressure distribution by 2 on 1.**




| **Depth X** | $\mathbf{k = \frac{7}{7+x}}$ | **ΔP (psf)** |
|---|---|---|
| 3.5 | 0.67 | 4000 |
| 7 | 0.50 | 3000 |
| 10.5 | 0.40 | 2400 |
| 14 | 0.33 | 2000 |
| 21 | 0.25 | 1500 |
| 28 | 0.20 | 1200 |
| 35 | 0.17 | 1000 |

**Step 2: Plot curve of $P_o$, $P_c$ and $P_F$ ($P_o + \Delta P$) vs. depth (use to obtain pressure of layer center).**




**Step 3: Compute settlement in each layer.**

Pier Settlement

- Sand Layer 4′ - 15′

$$\Delta H = H\frac{1}{C'}\text{Log}\frac{P_F}{P_0}$$

N′avg = 36

C′ = 90

$$\Delta H = 11\frac{1}{90}\text{Log}\frac{4350}{1050}(12)$$

$\Delta H = 0.90''$

- Clay Layer 15′ - 32′

$P_0 \rightarrow P_C$ (Preconsolidated)

$$\Delta H = H \frac{C_R}{1+e_0} \text{Log} \frac{P_F}{P_0}$$

$$\Delta H = 17 \frac{0.035}{1+0.97} \text{Log} \frac{3520}{1900} (12)$$

$\Delta H = 0.97''$

- Clay Layer 32′ - 40′

$P_0 \rightarrow P_C$ (Preconsolidated)

$$\Delta H = H \frac{C_R}{1+e_0} \text{Log} \frac{P_c}{P_0}$$

$$\Delta H = 8 \frac{0.035}{1+0.97} \text{Log} \frac{3450}{2710} (12)$$

$\Delta H = 0.18''$

$P_c \rightarrow P_F$ (Not Preconsolidated)

$$\Delta H = H \frac{C_c}{1+e_0} \text{Log} \frac{P_F}{P_c}$$

$$\Delta H = 8 \frac{0.35}{1+0.97} \text{Log} \frac{3840}{3450} (12)$$

$\Delta H = 0.80''$

| **Total Settlement** | |
|---|---|
| Sand | 0.90 |
| Clay | 0.97 |
| | 0.18 |
| | 0.80 |
| $\Delta H =$ | 2.85″ |

**Obtain Time-Settlement Relationship**

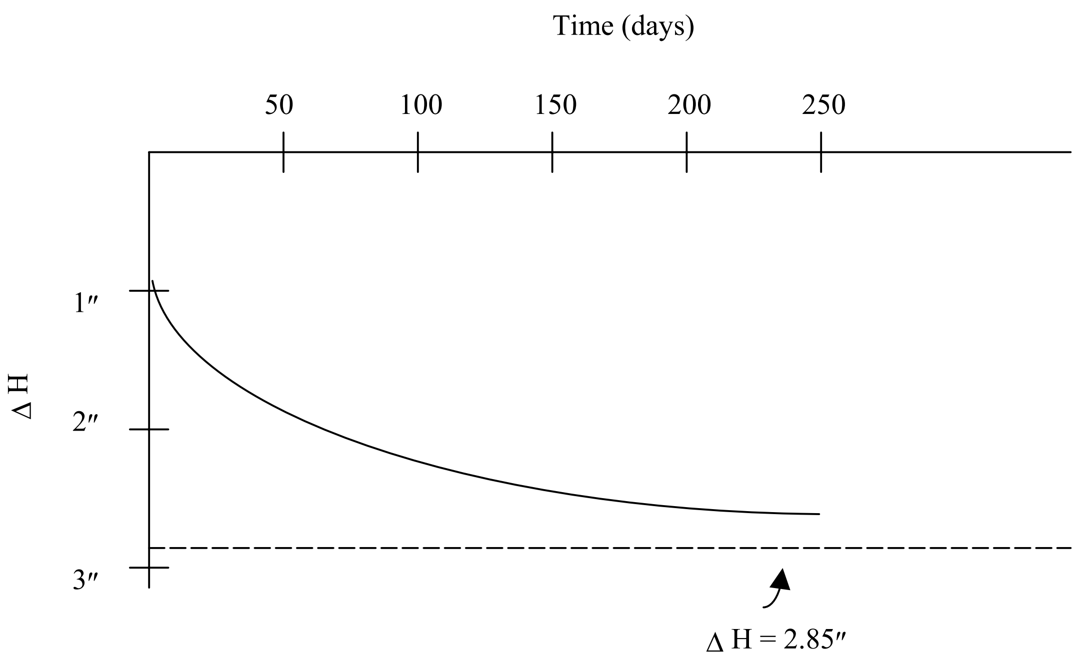
**Step 1: Obtain time for various settlement percentages.**

$t = \frac{T H_V^2}{C_V}$ (for Clay; Sand occurs immediately)

(Refer to Apple Freeway Design Example Chapter 6).

| % Consol. Clay | ΔH (inch) | T | $\frac{H_V^2}{C_V}$ | $t_{days}$ |
|---|---|---|---|---|
| 20 | 0.39 | 0.031 | 260 | 8 |
| 50 | 0.98 | 0.197 | | 51 |
| 70 | 1.37 | 0.408 | | 106 |
| 90 | 1.76 | 0.848 | | 220 |

**Step 2: Plot time-settlement curve for the pier.**



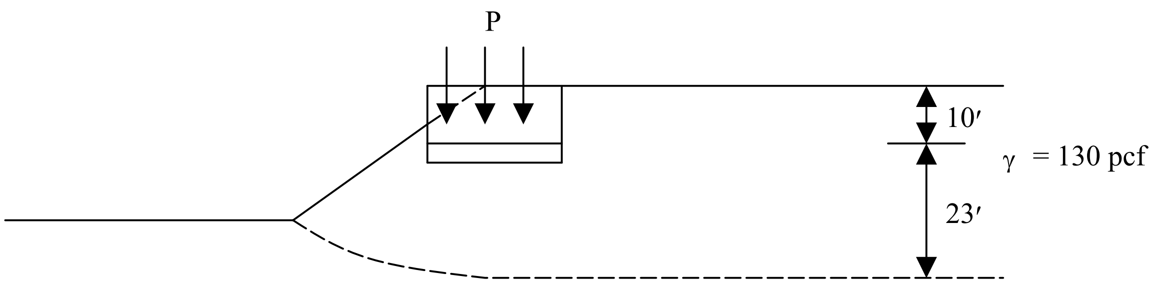

**East Abutment Footing Settlement.**

Assumptions:

- Abutment footing 10′ below top of fill
- Footing width = 7′

- Footing design pressure = 6300 psf ($\approx$ 3 TSF)
- No internal embankment consolidation
- Organic layer excavated

**Compute abutment-footing settlement.**




**Step 1: Obtain net footing pressure.**

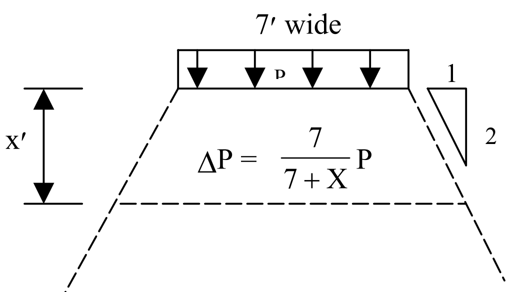
P = The net pressure applied at footing level assuming abutment constructed after embankment

P = 6300 psf – 1300 psf (Soil removed after waiting period)

P = 5000 psf

**Step 2: Determine pressure distribution.**

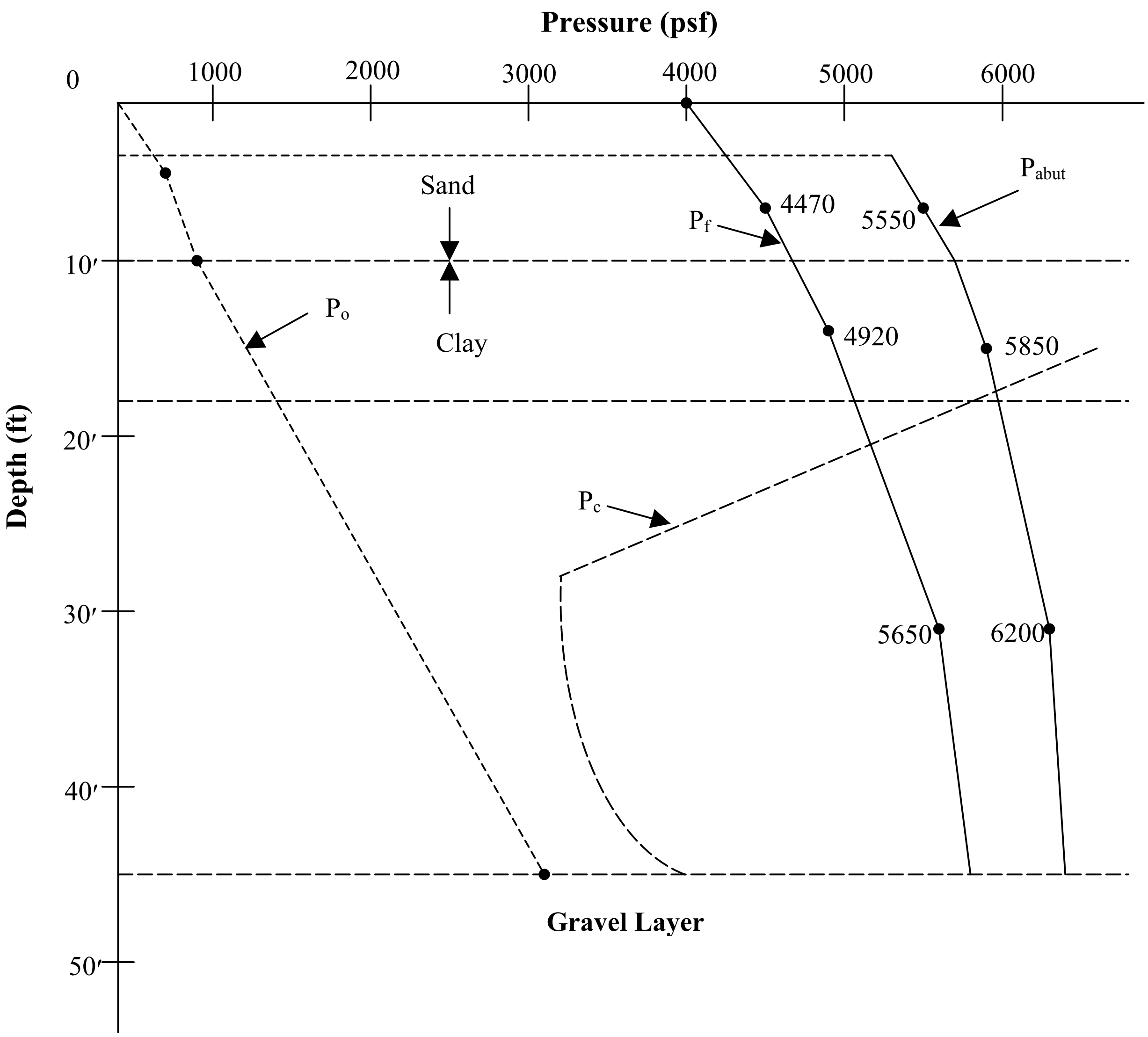
By 2 on 1 method – abutment




$$\Delta P = \left(\frac{7}{7+x}\right) 5000$$

| X Feet | ΔP psf |
|---|---|
| 23 (Ground Surface) | 1160 |
| 30 | 945 |
| 40 | 740 |
| 50 | 610 |
| 60 | 520 |

$P_F + \Delta P_{PSF} = P_{ABUT}$




**Step 3: Compute settlement in each layer.**

Abutment Settlement

- Layer 2 – Sand 3′ - 10′

$$\Delta H = H\frac{1}{C'}\text{Log}\frac{P_{ABUT}}{P_F}$$

$$\Delta H = 7\frac{1}{90}\text{Log}\frac{5550}{4470}$$

$\Delta H = 0.0065' \sim 0.08''$

- Layer 3 – Clay

10′ - 17′ (All Preconsolidated)

$$\Delta H = H\frac{C_R}{1+e_0}\text{Log}\frac{P_{ABUT}}{P_F}$$

$$\Delta H = 7\frac{0.035}{1+0.97}\text{Log}\frac{5850}{4920}$$

$\Delta H = 0.009' \sim 0.11''$

17′ - 45′ (Not Preconsolidated)

$$\Delta H = H\frac{C_c}{1+e_0}\text{Log}\frac{P_{ABUT}}{P_F}$$

$$\Delta H = 28\frac{0.35}{1+0.97}\text{Log}\frac{6200}{5650}$$

$\Delta H = 0.20' \sim 2.40''$

| Total East Abutment Settlement | |
|---|---|
| Layer 2 | 0.08 |
| Layer 3 | 0.11 |
| | 2.40 |
| $\Delta H_{ABUT} =$ | 2.59″ |

**Step 4: Determine time for settlement to occur.**

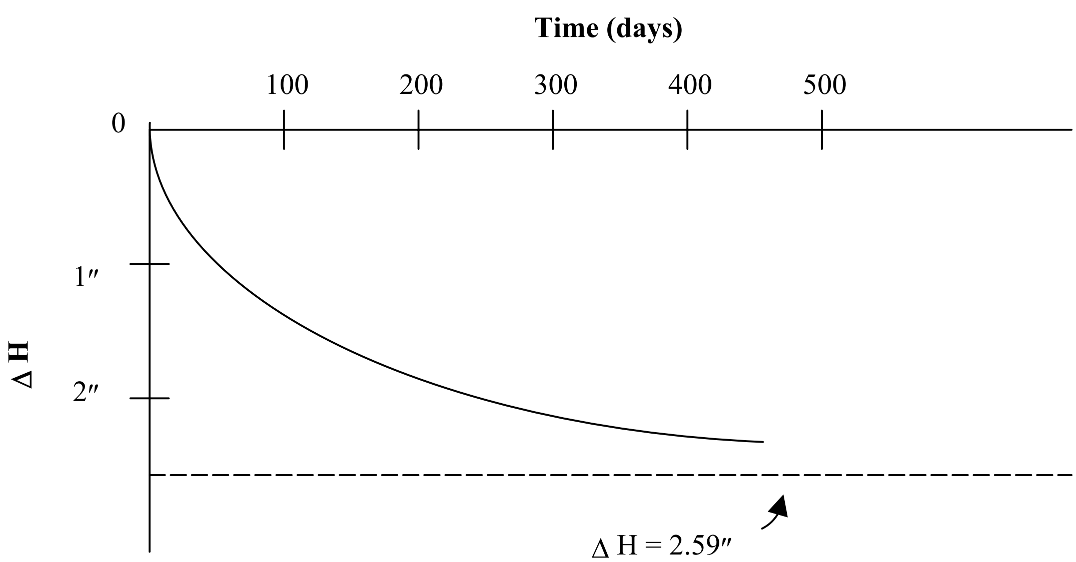
Time for settlement to occur

$$t = \frac{T H_V^{\ 2}}{C_V}$$

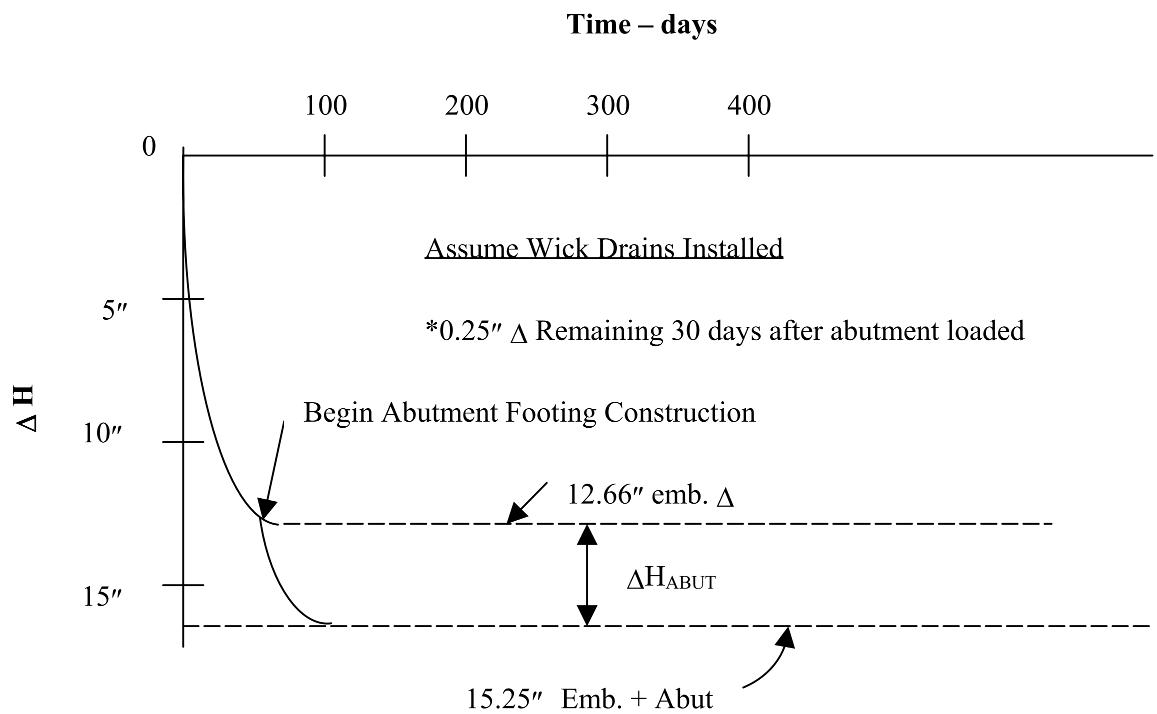
$H_V = 17.5'$

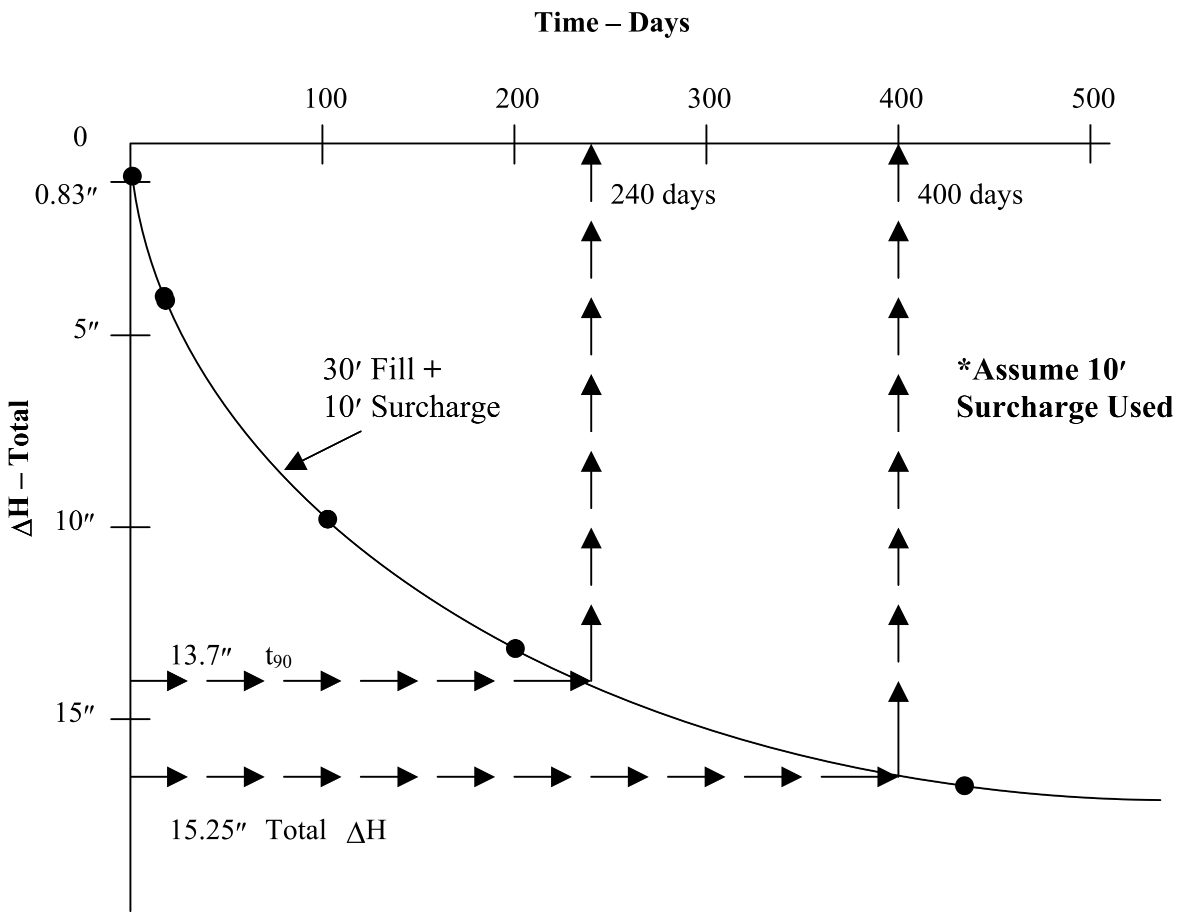
$C_V = 0.6\ ft^2/day$

| % Consol. Layer 3 | $\Delta$H (inches) | T | $\frac{H_V^{\ 2}}{C_V}$ | $t_{days}$ |
|---|---|---|---|---|
| 20 | 0.50 | 0.031 | 510.4 | 16 |
| 50 | 1.25 | 0.197 | | 101 |
| 70 | 1.76 | 0.403 | | 206 |
| 90 | 2.23 | 0.848 | | 433 |




Actual Abutment settlement will include settlement remaining due to embankment load. However surcharges and/or drains can be used to consolidate the clay layer for the embankments load and the abutment load.


Time – days
100
200
300
400
0
Assume Wick Drains Installed
5″
*0.25″ Δ Remaining 30 days after abutment loaded
Δ H
Begin Abutment Footing Construction
10″
12.66″ emb. Δ
ΔH_ABUT
15″
15.25″ Emb. + Abut




* Surcharge must be left in place for 13 months to dissipate all embankment and abutment ΔH

**Summary of the Spread Footing Design Phase for Apple Freeway Design Problem**

- Design Soil Profile

  Strength and consolidation values selected for all soil layers. Footing elevation and width chosen.

- Pier Bearing Capacity

  $Q_{allowable}$ = 3 tons/sq.ft.

- Pier Settlement

  Settlement = 2.8", $t_{90}$ = 220 days.

- Abutment Settlement

  Settlement - 2.6", $t_{90}$ = 433 days.

- Vertical Drains

  $t_{90}$ = 60 days - could reduce settlement to 0.25" after abutment constructed and loaded.

- Surcharge

  10' surcharge: $t_{90}$ = 240 days
  before abutment constructed.